Chapter 7
Evaluating and Improving Stability of Existing Structures

7-1. General

The stability of existing structures is sometimes reevaluated due to project modifications, changes in site conditions, improved knowledge of site data, or changes in stability criteria (ER 1110-2-1155). Since modifications to improve stability are often expensive, each structure should undergo a systematic, phased evaluation process to determine whether remediation is necessary. To avoid unnecessary modifications, all types of resisting effects should be considered. These include vertical friction, side friction, or three-dimensional effects. It may not be necessary to improve an existing structure's stability to satisfy the requirements for new structures, especially when the structure's remaining life is short or when there are no indications of any stability problem. Where current stability requirements cannot be satisfied, waivers may be obtained subject to consultation with and approval by CECW-ED.

7-2. Procedures

The following procedures are to be used in the evaluation of existing structures and in the determination of necessary corrective measures to improve stability. The procedures should be considered as guides and are not intended to replace engineering judgement by the project engineers. The stability condition should always be reviewed when there are significant changes in the loading conditions, severe damage due to accident, aging, or deterioration, discovery of structural deficiencies, revision of stability criteria to become more conservative, or when required by ER 1110-2-100. The phases listed below shall be followed in sequence until the prescribed performance objectives described in Chapter 3 are satisfied. Sliding and flotation factors of safety, by use of a well-defined site-information factor, need only be 90-percent of the safety factors required for new structures (Chapter 3). The 10-percent reduction, however, can only be used under the conditions specified in paragraph 3-2*c*.

a. PHASE I; Preliminary analysis and evaluation. Preliminary analysis should be performed based upon available data and actual conditions of the structure. Before performing the analysis, collect and review all the available data and information for the structure including geologic and foundation data, design plans, as-built plans, periodic inspection reports, damage reports, plans of previous modifications to the structure, measurements of movement, instrumentation data, and other pertinent information. It may be necessary for the engineer to inspect and examine the existing structure to assess its condition. The first step in evaluating the stability of an existing structure is to perform a preliminary two-dimensional stability analysis. Uplift pressures, the angle of internal friction, and cohesion should be evaluated using parametric studies to assess the impact of each parameter on the factors of safety. If the results of the Phase I preliminary analysis indicate that the structure meets the safety and performance objectives of Chapter 3, no further investigations are required. Otherwise, various options to meet safety and performance objectives should be investigated.

b. PHASE II; Study, investigation, and comprehensive analyses. When the preliminary analysis indicates the structure does not meet safety and performance objectives, a meeting should be arranged to decide on plans for the proposed comprehensive analysis, and to define the extent of the explorations and testing program needed to accurately define foundation strength parameters, the extent of the investigation and analytical program needed to accurately define the loading conditions, the remedial schemes to be studied, and the extent of any additional parametric studies. This meeting should include representatives from the district, division, CECW-ED, CECW-EG, and CECW-O. An exploration, sampling, testing, and instrumentation program should be developed, if needed, to determine the magnitude and the reasonable range of variation for the parameters which have significant effects on the safety and performance of the structure as determined by parametric studies. Testing laboratories should be used to the maximum extent practicable to develop shear strength data for the foundation and backfill materials.

Comprehensive stability analyses should be performed using the material properties and strength information obtained from the sampling and testing program. Consideration may be given to reducing backfill pressures to active-state conditions provided the displacements required to reach an active-state condition meet the performance objectives in Chapter 3. Remedial measures will not be required to improve stability should the parametric studies conducted in Phase II indicate the safety and performance objectives of Chapter 3 can be met without remediation.

c. PHASE III; Advanced analytical studies and reliability analyses.

(1) Advanced analytical studies. If the stability of the structure is still in question after completing preliminary and comprehensive analyses, advanced analytical studies should be performed. These studies should use two- and three- dimensional finite element methods to capture the interaction between the foundation, backfill, and the structure, and to capture the capacity of the structural system to distribute loads to adjacent monoliths and abutments. A fracture mechanics analysis (Dewey, Reich, and Saouma 1994) may be necessary to evaluate cracking and the progressive development of uplift pressures along potential failure planes. However, nonlinear analyses are not simple to perform, and fracture mechanics computer programs are not readily available. Fracture mechanics analysis should only be undertaken in special circumstances when approved by CECW-ED.

(2) Reliability analyses. Reliability analyses provide a reliable means of evaluating the stability of existing structures. This process combines what is known about a structure, with reasonable limits for the unknowns, to aid the engineer in assessing in probabilistic terms the potential for a stability failure. Although generally used for making investment decisions about projects competing for major rehabilitation funds, the process can be used to evaluate the risks associated with structures that, when analyzed by conventional deterministic procedures, indicate a stability-failure potential.

(3) Vertical shear (drag). Many existing gravity earth-retaining structures have performed satisfactorily for many years although they do not meet stability criteria. A research program investigation performed as a part of the Repair, Evaluation, Maintenance and Rehabilitation Program (REMR) was undertaken to study the stability of existing structures. The results of the REMR program and the experience from subsequent research programs are provided by Ebling, Duncan, and Clough (1990) and Filz and Duncan (1992). This research demonstrated that for rigid gravity walls on rock, where the rotation of the walls is not sufficient to fully mobilize the shear resistance of the backfill, a vertical shear force (drag) will develop as a result of vertical settlement within the backfill due to self weight. This shear force helps stabilize gravity walls founded on rock and should be considered when evaluating existing structures. The vertical shear force (drag) can be determined by a soil-structure interaction analysis using nonlinear finite element programs such as SOILSTRUCT as part of the Phase III analysis. Additional information and details on the use of vertical shear (drag) in stability analysis is presented in Appendix F.

7-3. Improving Stability

Stability can be improved through the use of foundation drains to reduce uplift, by adding anchors, and by adding concrete mass, concrete buttresses, or buttressing embankments.

a. Reducing uplift pressures. In most cases, the pressure reduction provided by foundation drains is much greater than that assumed for design. However, experience has shown that drains can clog, and this sometimes is temporarily unnoticed. If drains are properly maintained and if foundation testing and flow analysis provide supporting justification, the drain effectiveness can be increased beyond 50 percent with approval from CECW-ED. This criterion will depend on the pool operational plan, instrumentation to verify and evaluate uplift assumptions, and an adequate drain maintenance program. When actual uplift pressures are known, these uplift pressures can be extrapolated to pool levels used for the stability evaluation. However, extrapolation of existing uplift readings at low-pool elevations for use under high-pool conditions must be done with caution and with full consideration of the affect joint aperture changes can have on high-pool uplift pressures and their distribution from upstream to downstream within the foundation. Joints in the foundation rock can open or close as pool conditions change and the joint aperture change may not be uniform from upstream to downstream. Also, joint aperture changes can result in something other than linear uplift pressure distribution from upstream to downstream within the foundation. For these reasons, the use of existing uplift pressures in stability analyses must be supported with uplift pressure readings that reflect uplift pressure changes that have occurred under various pool levels and various climatic conditions. Information on the influence that geology, foundation treatment, drainage, changes in reservoir elevation, and seasonal temperature variations have on uplift pressures can be found in EPRI (1992). Flood-pool conditions that cause tension at the heel of the structure must be checked to determine if a crack will develop resulting in full uplift pressures in the crack. Uplift records must be sufficient to adequately define uplift distributions across the base from headwater to tailwater, and cover as many years as necessary to discern if there is a potential for any uplift pressure changes with time, and to provide uplift pressure information under various pool conditions. In existing gravity dams and other structures with drainage systems, generally the most economical way of improving stability is to clean out existing drains or to add new foundation drains. For structures without drainage systems, the uplift pressures can be reduced through the use of impervious blankets and cutoffs, although these methods of uplift reduction are much less reliable, and require ongoing monitoring to ensure continued performance of the cutoff.

b. Structural anchor systems. Tensioned vertical or inclined anchors can be installed to improve sliding stability. Vertical anchors installed across potential failure planes improve the sliding resistance by adding a normal force that increases the shear-friction component of the shear resistance. Inclining the anchors in the upstream direction will provide a horizontal component that can be used for additional shear resistance. However, untensioned anchors should not be used to improve sliding stability since they may require excessive displacement before becoming effective. Anchor systems are covered in detail in Chapter 8.

c. Adding concrete and buttressing to improve stability. Adding weight to a structure will increase compressive stress on failure planes and, thereby, increase the shear-frictional resistance to sliding. Concrete can be added to the front face, back face, or on top of the structure to increase the weight. The structure can also be buttressed with concrete blocks or with soil to improve sliding resistance. Often concrete blocks are placed against the toe of the structure and anchored with stressed tendons or bars (tensioned structural anchors) to further increase sliding resistance.

7-4. Case Histories

a. Eisenhower and Snell locks. Eisenhower and Snell locks are located on the St. Lawrence Seaway. The Corps of Engineers designed and constructed the two locks for the St. Lawrence Seaway Commission in the 1950s. Since that time, Corps personnel participated in annual inspections of the two locks. Cracking had been observed in the locks' monoliths. Advanced analytical studies were required to determine the adequacy of the existing locks' monoliths in terms of strength and stability, and to determine the advisability of lock monolith rehabilitation. A nonlinear finite element analysis, where loads are applied incrementally to simulate the actual field conditions, was used to evaluate monolith performance. Six categories were modeled in the FEM analysis:

(1) Initial conditions after excavation.

(2) Placement of the lock wall.

(3) Raising of the water table in the backfill.

(4) Formation of monolith cracks.

(5) Placement of tensioned structural anchors within the monoliths.

(6) Operation of the lock after rehabilitation.

The refined analysis procedure demonstrated the lock monoliths, once rehabilitated by installing tensioned structural anchors within the monoliths, could operate safely without structural or stability problems. This could not have been accomplished using traditional stability analysis procedures. (Mosher, Bevins, and Neeley 1991).

b. Lock 27 study. Lock 27 is located on the Mississippi River. Monolith 7E is a gravity earth-retaining structure that when evaluated using conventional equilibrium-based analysis methods demonstrated there was crack propagation at the structure-foundation interface. The crack propagation was evaluated using a finite element analysis procedure, a fracture-mechanics discrete crack analysis procedure, and a fracture-mechanics smeared crack analysis procedure. The finite element analysis utilized the SOILSTRUCT- Alpha procedure described in Chapter 6 above. The special-purpose FEM code MERLIN was used for the discrete crack analysis procedure, and the special-purpose FEM code CG-DAMS was used for the smeared crack analysis procedure. The crack propagation length at the base of Monolith 7E was similar for each of the three advanced analytical methods and significantly less than that predicted by the conventional equilibrium-based analysis method. Base pressure distribution was also significantly different from the linear distribution assumed for the traditional method of analysis (Ebeling 1996).

c. Stewart Mountain Dam. Stewart Mountain Dam is a thin arch dam built in 1929 to 1930. An evaluation of the dam revealed that it was not dynamically stable during a Maximum Credible Earthquake (MCE) event; therefore, to stabilize the dam, 62 tensioned, structural anchors on 2.6m (8.5ft) to 3.0m (10.0ft) center-to-center spacing were installed from the crest of the dam and generally extended into the foundation. The bearing plates were designed to withstand a 4,450kN (1,000k) tension load. A new overlay of 35mP (5,000psi) compressive strength concrete was placed on the crest of the dam to withstand punching shear. This overlay was designed to act as a beam on a flexible foundation because of the differences in the modulus of elasticity of the new overlay concrete and that of the existing dam concrete. A two-dimensional ADINA finite-element study was performed on a typical localized cross section of the top of the dam. The bearing plate, overlay, and existing dam concrete were modeled. Reinforcing for the overlay was designed to withstand the computed horizontal tension stresses under the bearing plate and the shear stresses below the edges of the bearing plate. For a detailed discussion of the design, refer to Bureau of Reclamation 1991.